Design/Detail Guidelines

Abstract

Attachment B presents guidelines on several issues encountered in retrofit or new seismic design/detailing of bridges. It is important to note that the subjects listed below can be read or referred to separately. These subjects include the following:

1. Steel column casing/Design and Details.
2. High strength fiber epoxy column casing/Design and Details.
3. Allowable column shear values inside and outside plastic hinge zone.
4. Design of cap beam and outrigger for torsion.
5. Pipe seat extender vertical and transverse capacities.
6. Column bar development length.
7. Column removal/replacement falsework design.
8. Design/Specification coordination issues.
    a. Welding Grade 60 to Grade 40 bars.
    b. Roughened concrete surface in shear friction design.
    c. Grouting of cored holes with inserted bolts.
    d. Column removal/replacement shoring.
    e. Removal and replacement of restrainers.
9. Footing retrofit considerations.
10. Pile foundations.
11. Pile extension bents.
12. Exposed bent caps.
13. Pier wall system transverse design.
14. Seismic anchor slabs.

1. Steel Column Casing/Design and Details

Steel jacketing tests at UCSD were initially performed on 40% scale models of circular and rectangular columns. These casings were designed with thicknesses to provide 300 psi confining pressure on the columns in the plastic hinge zones. When columns with poor details (minimal confinement steel and lap splices at the footings) were retrofitted with shells that provided 300 psi confining pressure, their performance improved dramatically. As a result, the steel jacket will be used as the standard retrofit detail on most projects. Other systems, such as high strength fiber epoxy casings and wire wrap casings, will be allowed as alternatives as they are developed.

Upon completion of the retrofit analysis, the designer must decide if column casings are required and which type to use. Basically, there are three types of casings. The type F shell provides a fixed end condition. The type P shell permits a pin to form by allowing lap splices to slip. The third type of casing is the P/F shell. The P/F shell is a full length shell that provides a fixed condition at the top of the column and has polystyrene at the lap splice to allow pin formation at the footing. When using steel casings, it is necessary to provide a minimum of 2" clearance between the casing ends and the soffit and/or footings. The gap prevents the casing from bearing on the attached member. Bearing would increase effects of the plastic moment and probably fail the footing or soffit. The gap is required for all casings which are fixed to columns by grout, therefore, the partial height type P casing is the only casing that would not require a gap.

Charts have been developed to give casing dimensions and thicknesses for common sizes of rectangular columns. These charts (Figures B3 and B4) give curve data used to produce the most efficient casing around the given column. In order to prevent possible construction claims, the curve data is **not** to be listed on the plans. This information is for design, detailing and estimation purposes only. The only dimensions that should be listed on the plans are the "x" and "y" dimensions as well as the casing thickness.

When determining casing thickness requirements, type F casings can be read directly from the charts shown in Figures B3 and B4. Note that casing thicknesses are not to exceed 1", in which case the designer is referred to note 5 on Figure B2. The column casing thicknesses for type F shells were developed using thin wall pressure element theory shown on Figure B1. The required shell thickness is directly related to the radius. For rectangular columns, the shell is made up of partial circles with two different radii. The designer may use the average of the two radii to determine the casing thickness. The designer should note that type P shells require a minimum thickness of ⅜".

The designer may encounter a situation where the charts will not be applicable. For example, the designer may need to provide more clearance or a shorter radius to reduce shell thickness requirements. In these situations, the designer can use the design formula for an ellipse given on Figure B2. The casing is then made from partial circular shapes that most closely matches the ellipse. For the casing thickness, the designer will use the formula on Figure B1.

At some point during the design process, the designer should coordinate with the specification writer on the following issues:

a. When the minimum spacing between the column and the casing is equal to or greater than ¾", the grout mix should contain pea gravel.

b. If a pea gravel grout is used for elliptical shells, injection ports may be needed on four sides because of restricted clearances at column corners. A similar detail may apply if elliptical jackets are used for rectangular columns with round ends and tight clearances.

c. In type P and P/F shells, the polystyrene insert should have a 12" gap at the vertical seam of the casings. This is to prevent the polystyrene from burning during the welding process.

d. For tall casings, some measures should be taken to prevent casing from bulging due to large hydrostatic head during the grouting operation. One solution is to pump the grout in lifts, allowing each preceding lift to set, to reduce the hydrostatic head. Another solution is to add temporary whaler around the casing to provide extra confinement and strength while pumping the grout.

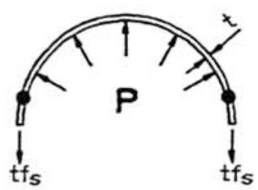
The column casing design aids attached (Figures B1 to B4), along with the column casing standard sheet, should cover most jacketing situations. For odd shaped columns or any situation where the design charts do not apply, the designer should use good engineering judgement. If necessary, the designer should consult with the design senior, the retrofit specialist and the SEITECH representative.

CASING THICKNESS:

TWO CONTROLLING PARAMETERS:

A) Thin Walled Pressure Element (TWPE)
B) University of California San Diego Tests (UCSD Test)

FROM TWPE:



$$\boxed{\frac{\sigma_{LONG}}{R_{LONG}} + \frac{\sigma_{TRAN}}{R_{TRAN}} = \frac{P}{T}}$$

FOR COLUMN CASING: $R_{LONG} \longrightarrow \infty$

$$\therefore \quad \frac{\sigma_{TRAN}}{R_{TRAN}} = \frac{P}{t}$$

NOTES:

σ_{LONG} = Sigma(stress) Longitudinally
σ_{TRAN} = Sigma(stress) Transversely
R_{LONG} = Radius Longitudinally
R_{TRAN} = Radius Transversly
P = Internal Pressure
t = Thickness of Material

FROM UCSD TEST:

At the point when a plastic hinge formed in the lap splice region, the strain in the steel casing was equal to 0.001 in/in. The steel casing must be designed such that it produces 300 psi of confining pressure at this measured strain.

$\therefore \quad f_s = \varepsilon_s E_s$ = 29,000 psi for lap-splice condition.

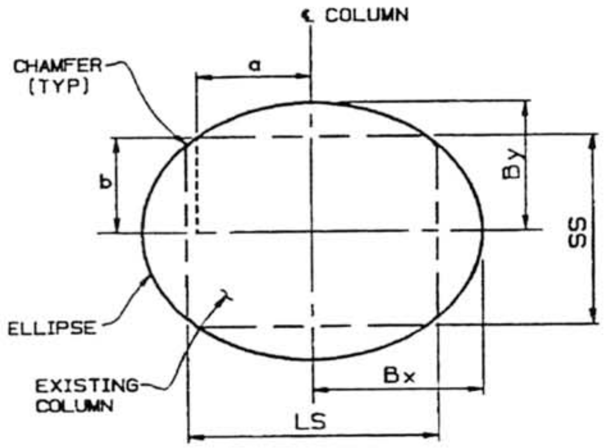
f_s = (Full Yield) = 36,000 psi for continous reinforcement.

$$t_{LAP\text{-}SPLICE} = \frac{\text{Radius (Average)}}{100}(12)$$

$$t_{CONT\ REINF} = \frac{\text{Radius (Average)}}{120}(12)$$

Elliptical Steel Casing Thickness Requirement for Plastic Hinge Zones

Figure B1



$$A_{SR} = \frac{LS}{SS}$$

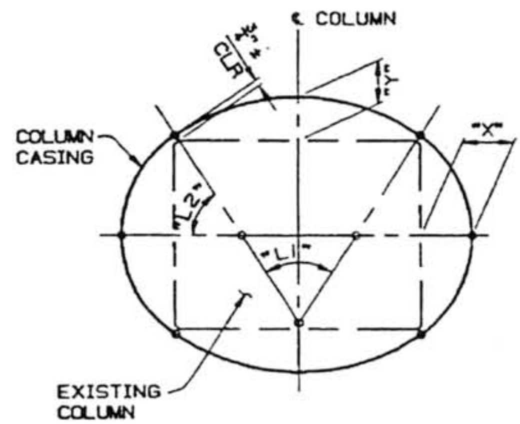
$$B_Y = \sqrt{b^2 + \frac{a^2}{(A_{SR})^2}}$$

$$B_X = B_Y \times A_{SR}$$

A_{SR} = ASPECT RATIO
LS = LONG SIDE
SS = SHORT SIDE

ELLIPSE GEOMETRY

NO SCALE



COLUMN CASING

NO SCALE

General Notes for Design & Analysis:

1. "X" & "Y" Dimensions are to be shown on the Contract Plans. See sheet 3 & 4 for location of "X & Y" dimensions.
2. Required casing thickness in the plastic hinge zone shall be the dimension t shown in the tables on the following pages.
3. Type "P" casing shall be $\frac{3}{8}$" thick unless otherwise noted on plans.
4. Maximum plate thickness shall be 1" and minimum plate thickness is $\frac{3}{8}$".
5. If 1" maximum is exceeded, use of anchor bolts, stiffening channels, etc, must be incorporated to adequately confine the columns.
6. UCSD Tests were concucted using a $20d_b$ lap length of 40ksi yield strength rebar.
7. The aesthetics section shall be consulted to obtain a workable and aesthetically pleasing solution when different plate thicknesses are joined, exterior stiffeners are attached or through bolts are installed.

Figure B2

COLUMN CASING DATA						CASING THICKNESS			
COLUMN SIZE	CURVE DATA (L1) / CURVE DATA (L2)					PLASTIC HINGE ZONE			
	RADIUS	DELTA	CURVE LENGTH	TANGENT	CHORD LENGTH	X*	Y*	$t^{**}_{LAP\ SPLICE}$	$t^{**}_{CONT\ REINF}$
2'-0"x3'-0"	2'-1 1/2"	59° 59' 00"	3'-1/8"	1'-8 1/2"	2'-1 1/2"	7 3/16"	5 5/8"	3/8"	3/8"
	1'-2 7/8"	60° 00' 30"	1'-3 5/8"	0'-8 5/8"	1'-2 7/8"				
2'-0"x4'-0"	5'-3/4"	45° 47' 04"	4'-1/2"	2'-1 5/8"	3'-1 1/4"	8 3/16"	5 11/16"	3/8"	3/8"
	1'-2"	67° 06' 28"	1'-4 1/2"	0'-9 1/4"	1'-3 1/2"				
2'-0"x5'-0"	7'-9 1/4"	36° 58' 14"	5'-1/8"	2'-7 1/8"	4'-1 1/8"	8 7/8"	5 3/4"	5/8"	1/2"
	1'-1 5/8"	71° 30' 53"	1'-5"	0'-9 3/4"	1'-4"				
2'-0"x6'-0"	11'-1"	30° 59' 00"	1'-5 5/8"	0'-10 1/4"	1'-4 3/8"	9 3/8"	5 13/16"	3/4"	5/8"
	1'-1 1/2"	74° 30' 30"	1'-5 5/8"	0'-10 1/4"	1'-4 3/8"				
2'-0"x7'-0"	15'-0"	26° 39' 26"	6'-1 1 3/4"	3'-6 5/8"	6'-11"	9 13/16"	5 7/8"	1"	7/8"
	1'-1 3/8"	76° 40' 17"	1'-5 7/8"	0'-10 5/8"	1'-4 5/8"				
2'-0"x8'-0"	19'-6 1/8"	23° 23' 14"	7'-1 5/8"	4'-1/2"	7'-10 7/8"	10"	5 13/16"	USE*** OTHER MEANS	1"
	1'-1 1/4"	78° 18' 23"	1'-6 1/8"	0'-10 3/4"	1'-4 3/4"				

COLUMN JACKET DATA						CASING THICKNESS			
COLUMN SIZE	CURVE DATA (L1) / CURVE DATA (L2)					PLASTIC HINGE ZONE			
	RADIUS	DELTA	CURVE LENGTH	TANGENT	CHORD LENGTH	X*	Y*	$t^{**}_{LAP\ SPLICE}$	$t^{**}_{CONT\ REINF}$
3'-0"x4'-0"	3'-7"	67° 34' 34"	4'-2 3/4"	2'-4 3/4"	3'-1 7/8"	10 1/16"	8 1/4"	3/8"	3/8"
	1'-10 7/8"	56° 12' 43"	1'-10 1/2"	1'-1/4"	1'-9 1/2"				
3'-0"x5'-0"	5'-4 1/2"	54° 52' 08"	5'-1 3/4"	2'-9 1/2"	4'-1 1/2"	11 3/16"	8 1/8"	1/2"	3/8"
	1'-9 5/16"	62° 33' 56"	1'-1 1/4"	1'-1"	1'-10 1/8"				
3'-0"x6'-0"	7'-7"	46° 07' 38"	6'-1/4"	3'-2 3/4"	5'-1 3/8"	1'-3/16"	8 3/16"	5/8"	1/2"
	1'-8 5/8"	66° 56' 11"	2'-1/8"	1'-5/8"	1'-10 5/8"				
3'-0"x7'-0"	10'-2 1/4"	39° 45' 48"	7'-7/8"	3'-8 1/4"	6'-1 1/8"	1'-13/16"	8 3/16"	3/4"	5/8"
	1'-8 1/8"	70° 07' 06"	2'-5/8"	1'-2 1/8"	1'-1 1/8"				
3'-0"x8'-0"	13'-2 1/2"	34° 55' 50"	8'-5/8"	4'-7/8"	7'-1 1/8"	1'-1/2"	8 5/16"	1"	3/4"
	1'-7 15/16"	72° 32' 05"	2'-1/4"	1'-2 5/8"	1'-1 5/8"				
3'-0"x9'-0"	16'-7 1/2"	31° 08' 17"	9'-3/8"	4'-7 5/8"	8'-1 1/8"	1'-2"	8 3/8"	USE*** OTHER MEANS	1"
	1'-7 13/16"	74° 25' 51"	2'-3/4"	1'-3"	2'-0"				
3'-0"x10'-0"	17'-5 9/16"	33° 00' 13"	10'-11/16"	5'2 1/16"	9'-1 1/16"	1'-1 11/16"	9 9/16"	USE*** OTHER MEANS	
	1'-7 3/4"	73° 29' 53"	2'-1 5/16"	1'-2 3/4"	1'-1 5/8"				
3'-0"x11'-0"	24'-8"	25° 34' 28"	11'-1/8"	5'-7 1/4"	10'-11"	1'-2 13/16"	8 1/2"	USE*** OTHER MEANS	
	1'-7 5/8"	77° 12' 46"	2'-2 1/2"	1'-3 5/8"	2'-1/2"				
3'-0"x12'-0"	29'-3 1/4"	23° 28' 30"	11'-1 7/8"	6'-1"	11'-10 7/8"	1'-2 7/8"	8 5/16"	USE*** OTHER MEANS	
	1'-7 7/8"	78° 15' 45"	2'-2 7/16"	1'-3 3/4"	2'-7/16"				

NOTES:

* DIMENSIONS TO BE SHOWN ON PLANS. DIMENSIONS SHOULD BE ROUNDED UP AS APPROVED BY DESIGNER.

** SHELL THICKNESS TO BE USED IN PLASTIC HINGE ZONES. FOR TYPE P CASING USE MIN t=3/8".

*** SEE NOTE 5 ON FIGURE B2.

Figure B3

COLUMN JACKET DATA						CASING THICKNESS			
COLUMN SIZE	CURVE DATA (L1) / CURVE DATA (L2)					PLASTIC HINGE ZONE			
	RADIUS	DELTA	CURVE LENGTH	TANGENT	CHORD LENGTH	X*	Y*	$t^{**}_{LAP\ SPLICE}$	$t^{**}_{CONT\ REINF}$
4'-0"x3'-0"	3'-7"	67° 34' 34"	4'-2 3/4"	2'-4 3/4"	3'-1 7/8"	10 7/16"	8 1/2"	3/8"	3/8"
	1'-10 7/8"	56° 12 43"	1'-10 1/2"	1'- 1/4"	1'-9 1/2"				
4'-0"x4'-0"	2'-10 1/2"	360° 00' 00"	18'- 3/4"	-	-	10 1/2"	10 1/2"	3/8"	3/8"
	-	-	-	-	-				
4'-0"x5'-0"	4'-2 3/4"	72° 06' 38"	5'-3 7/8"	3'- 7/8"	4'-1 7/8"	1'- 1/2"	10 9/16"	1/2"	3/8"
	2'-6 3/4"	53° 56' 41"	2'-4 7/8"	1'-3 5/8"	2'-3 7/8"				
4'-0"x6'-0"	5'-10 1/2"	60° 52' 26"	6'-2 7/8"	3'-5 3/8"	5'-1 3/8"	1'- 7/8"	10 1/2"	1/2"	1/2"
	2'-4 3/4"	59° 33' 47"	2'-5 7/8"	1'-4 3/8"	2'-4 1/2"				
4'-0"x7'-0"	7'-10"	52° 36' 43"	7'-2 1/4"	3'-10 1/2"	6'-1 1/4"	1'-3 1/16"	10 9/16"	5/8"	1/2"
	2'-3 3/4"	63° 41' 38"	2'-6 3/4"	1'-5 1/4"	2'-5 1/4"				
4'-0"x8'-0"	10'-1 1/4"	46° 17' 51"	8'-2"	4'-3 7/8"	7'-1 3/8"	1'-4 1/8"	10 3/4"	3/4"	5/8"
	2'-3 3/16"	66° 51' 05"	2'-7 3/4"	1'-6"	2'-6"				
4'-0"x9'-0"	12'-8"	41° 19' 16"	9'- 5/8"	4'-9 1/4"	8'-1 1/4"	1'-4 15/16"	10 13/16"	1"	3/4"
	2'-2 3/4"	69° 20' 22"	2'-8 3/8"	1'-6 1/2"	2'-6 1/2"				
4'-0"x10'-0"	15'-6 1/4"	37° 18' 08"	10'-1 1/4"	5'-2 7/8"	9'-1 1/8"	1'-5 7/16"	10 11/16"		1"
	2'-2 5/16"	71° 20' 56"	2'-8 3/4"	1'-7"	2'-6 3/4"				
4'-0"x11'-0"	18'-8 1/4"	33° 59' 24"	11'-1"	5'-8 1/2"	10'-1 1/8"	1'-6"	10 3/4"	USE*** OTHER MEANS	
	2'-2 1/8"	73° 00' 18"	2'-9 1/4"	1'-7 1/4"	2'-7 1/8"				
4'-0"x12'-0"	22'-2"	31° 12' 54"	12'- 7/8"	6'-2 3/8"	11'-1 1/8"	1'-6 5/8"	10 15/16"		
	2'-2 1/8"	74° 23' 33"	2'-10"	1'-7 7/8"	2'-7 5/8"				
4'-0"x13'-0"	25'-11"	28° 51' 52"	13'- 5/8"	6'-8"	12'-11"	1'-6 13/16"	10 11/16"		
	2'-1 11/16"	75° 34' 04"	2'-9 7/8"	1'-7 7/8"	2'-7 1/2"				
4'-0"x14'-0"	30'-0"	26° 49' 41"	14'- 5/8"	7'-1 7/8"	13'-1 1/8"	1'-7 5/16"	10 15/16"		
	2'-1 13/16"	76° 35' 10"	2'-10 1/2"	1'-8 3/8"	2'-8"				
4'-0"x15'-0"	34'-4 1/4"	25° 03' 53"	15'- 3/8"	7'-7 5/8"	14'-10 15/16"	1'-7 7/16"	10 3/4"		
	2'-1 1/2"	77° 28' 03"	2'-10 1/2"	1'-8 7/16"	2'-7 15/16"				
4'-0"x16'-0"	39'- 1/2"	23° 31' 06"	16'- 1/4"	8'-1 1/2"	15'-11"	1'-7 7/8"	10 15/16"		
	2'-1 5/8"	78° 14' 27"	2'-11"	1'-8 7/8"	2'-8 3/8"				

NOTES:

* DIMENSIONS TO BE SHOWN ON PLANS. DIMENSIONS SHOULD BE ROUNDED UP AS APPROVED BY DESIGNER.

** SHELL THICKNESS TO BE USED IN PLASTIC HINGE ZONES. FOR TYPE P CASING USE MIN $t = \frac{3}{8}$".

*** SEE NOTE 5 ON FIGURE B2.

Figure B4

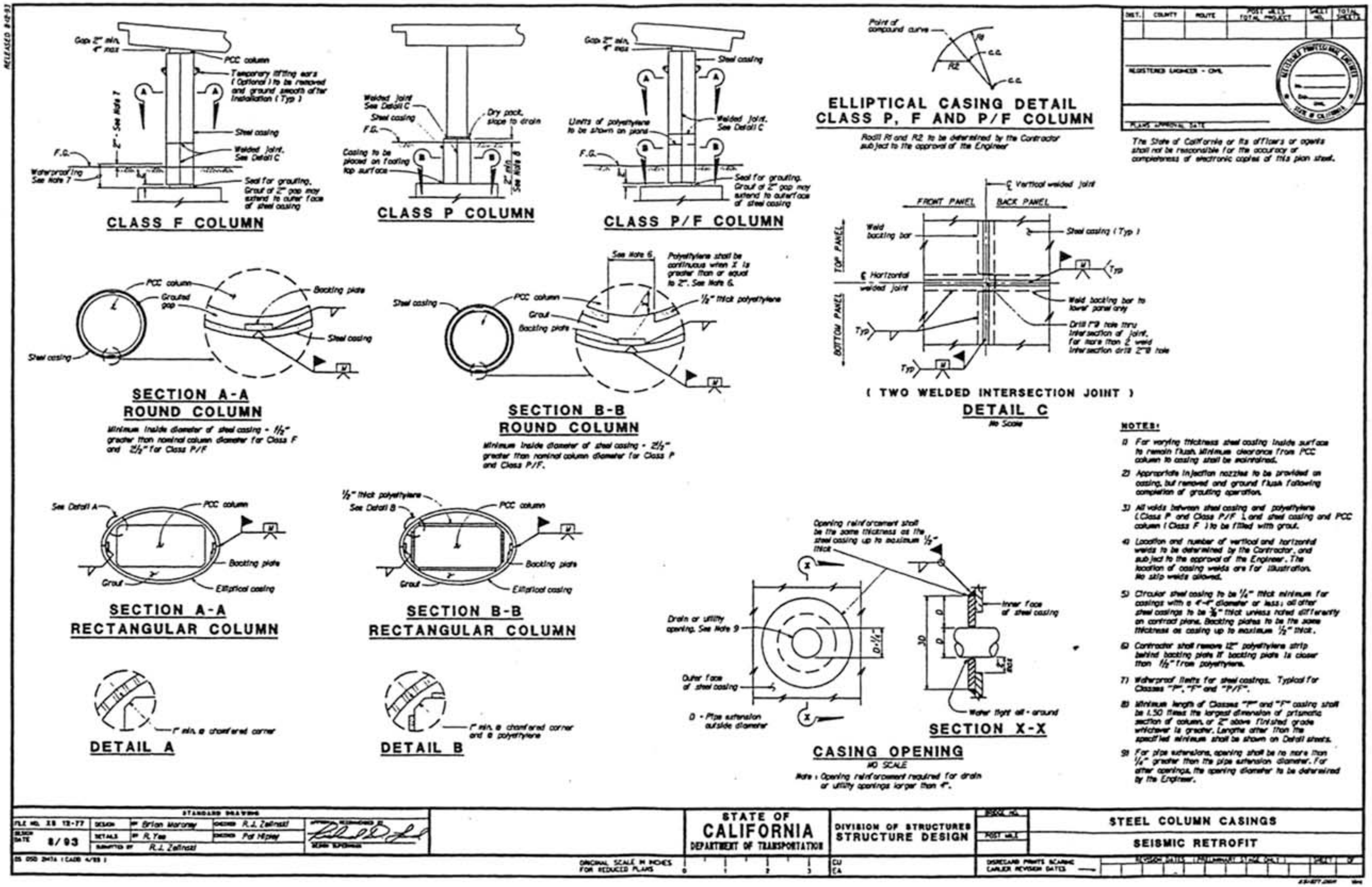

CLASS F COLUMN
CLASS P COLUMN
CLASS P/F COLUMN
ELLIPTICAL CASING DETAIL
CLASS P, F AND P/F COLUMN
SECTION A-A
ROUND COLUMN
SECTION B-B
ROUND COLUMN
(TWO WELDED INTERSECTION JOINT)
DETAIL C
SECTION A-A
RECTANGULAR COLUMN
SECTION B-B
RECTANGULAR COLUMN
DETAIL A
DETAIL B
CASING OPENING
NO SCALE
SECTION X-X
NOTES:
STATE OF
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
STEEL COLUMN CASINGS
SEISMIC RETROFIT

Figure B5

2. *Composite Column Casings*

Several composites column casing systems have undergone laboratory testing and are approved for use in limited situations. Composite column casing thicknesses as shown on the Standard Drawing are designed to prevent plastic shearing. Material testing standards and provisional specifications have been developed to allow limited field installations for both E-glass and carbon fiber composites, under strict conditions.

Composites systems shall be specified as an alternative if conditions below are satisfied:

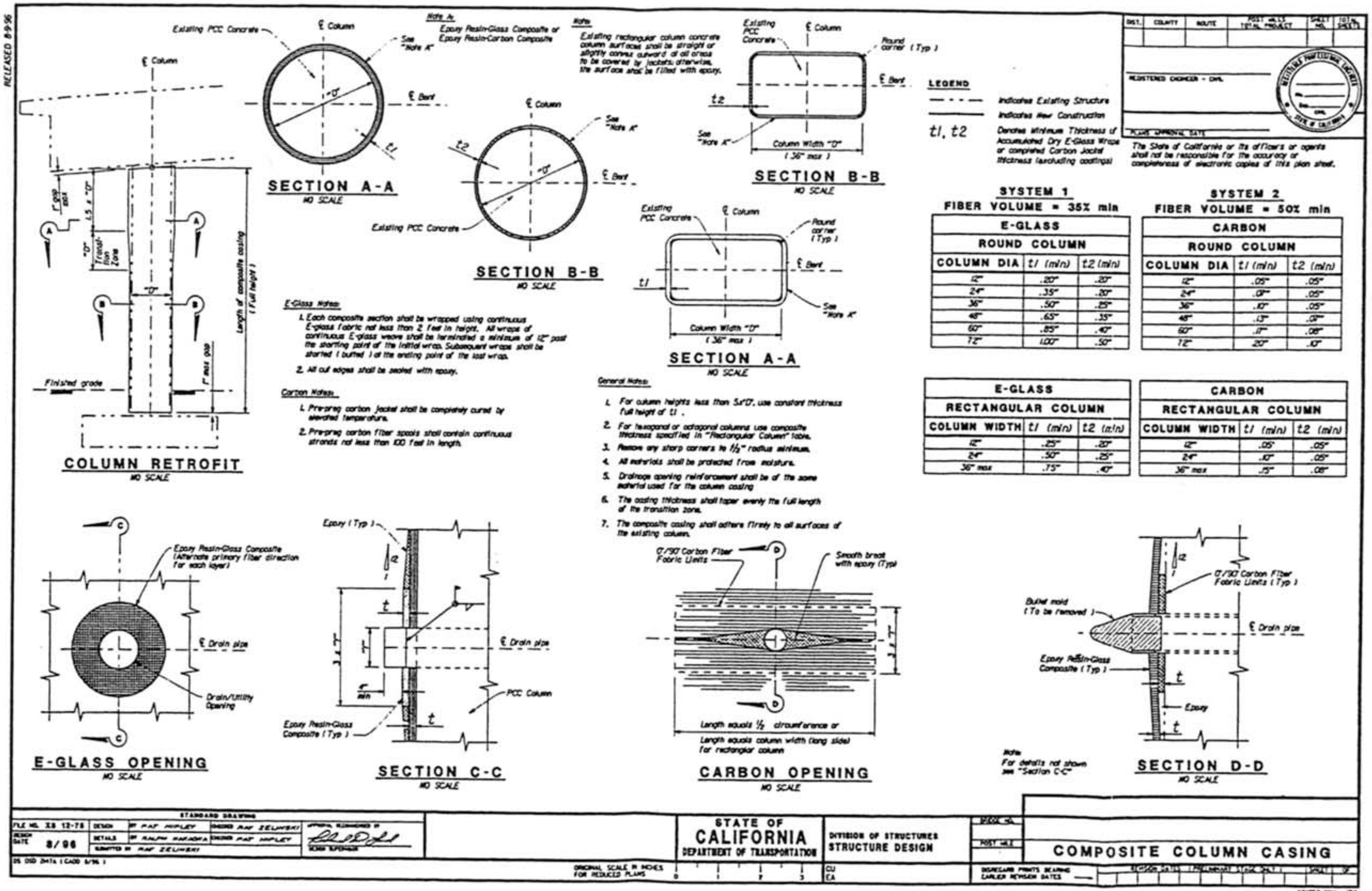
1. In all cases, all projects shall be detailed for steel casings as a standard with composites retrofit as an alternative.
2. Displacement ductility demand not more than 6 for circular columns and not more than 3 for rectangular columns. It may be permissible to use composites on circular columns with ductility demands approaching 8, with the written approval of the Office of Earthquake Engineering and the Design Supervisor.
3. For rectangular columns, the longest dimension is limited to a maximum of 36 inches. Rectangular column sides aspect ratio shall not be greater than 1.5.
4. For circular columns, the diameter must be 72 inches or less.
5. A steel jacket is the only approved retrofit method for columns that require a fully contained (fixed) lap splice. Composites may be used if a pin or slipping is assumed in the analysis at a lap splice.
6. Composites shall not be used for single column bent structures.
7. Composites shall not be used if the axial dead load is greater than $0.15 f'_c Ag$.
8. Composites shall not be used if the columns longitudinal reinforcement ratio is greater than 2.5%.
9. Composites shall not be used for bridges which require flame-sprayed plastic.
10. Composites shall be used with prismatic columns only.

For situations not falling within the above limits, the Office of Earthquake Engineering shall be consulted for necessary design guidelines and approval. A list of current allowable systems may be obtained from the Office of Earthquake Engineering, New Technology Management Branch at (916) 227-8247. Requirements above are subject to change as more information becomes available.

Questions on the above should be directed to the New Technology Management Branch at (916) 227-8247 or Seismic Technology at (916) 227-8806.

Design Instructions

Refer to the attached detail sheet titled "Composite Column Casing" (Figure B6) for design instructions.



SYSTEM 1
FIBER VOLUME = 35% min

E-GLASS ROUND COLUMN		
COLUMN DIA	t1 (min)	t2 (min)
12"	.20"	.20"
24"	.35"	.20"
36"	.50"	.25"
48"	.65"	.35"
60"	.85"	.40"
72"	1.00"	.50"

SYSTEM 2
FIBER VOLUME = 50% min

CARBON ROUND COLUMN		
COLUMN DIA	t1 (min)	t2 (min)
12"	.05"	.05"
24"	.07"	.05"
36"	.10"	.05"
48"	.13"	.07"
60"	.17"	.08"
72"	.20"	.10"

E-GLASS RECTANGULAR COLUMN		
COLUMN WIDTH	t1 (min)	t2 (min)
12"	.25"	.20"
24"	.50"	.25"
36" max	.75"	.40"

CARBON RECTANGULAR COLUMN		
COLUMN WIDTH	t1 (min)	t2 (min)
12"	.05"	.05"
24"	.10"	.05"
36" max	.15"	.08"

Figure B6

3. *Allowable Column Shear Values Inside and Outside Plastic Hinge Zone*

Allowable shear strength in existing columns shall be calculated based on the following relationship:

$V_n \quad = V_c + V_t =$ shear carried by concrete + shear carried by truss mechanism.

$V_n \quad = \vartheta_c A_e + A_v f_{yt}\, d/s$ for rectangular sections

and

$V_n \quad = \vartheta_c A_e + \pi/2\, A_s f_{yt}\, D'/s$ for circular sections

where

A_e : effective shear area taken as 0.8 Ag (Ag = gross section area)

A_v : total cross-sectional area of transverse reinforcement within spacing s.

f_{yt} : probable yield strength of transverse reinforcement

d : effective depth of column

s : spacing of transverse reinforcement

A_s : cross-sectional area of transverse reinforcement (hoop or spiral)

D' : hoop or spiral diameter

ϑ_c : concrete shear stress is dependent on displacement ductility demand ratio and net compressive axial stress.

$\vartheta_c \quad = \text{Factor 1} \times \text{Factor 2} \times \sqrt{f'_c} \le 4\sqrt{f'_c}$

where

f'_c : aged concrete strength

Factor 1 can be interpolated between curves given for $\rho'' f_{yt}$ equal to 50 and 350 psi, as shown in Figure B7. Note that factor 1 need not be taken less than 0.3. The interpolation equation is given by:

$$Factor\ 1 = \frac{\rho'' f_{yt}}{150} + 3.67 - \mu_\Delta \leq 3.0$$

where

$$\rho'' = \frac{volume\ of\ transverse\ reinforcement}{volume\ of\ column\ core}$$

Note that for a circular section this equation reduces to:

$$\rho'' = \frac{4 \times (Area\ of\ spiral)}{D' \times s}$$

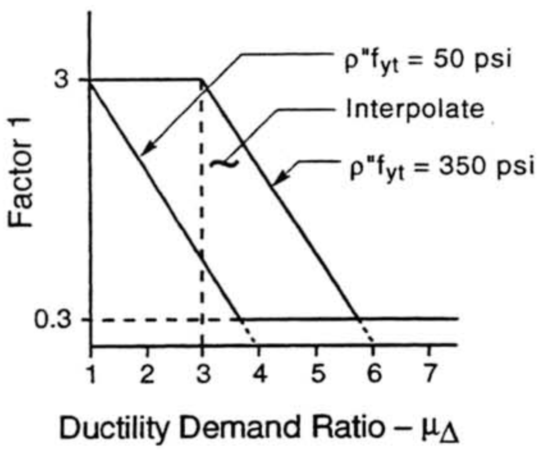


Figure B7

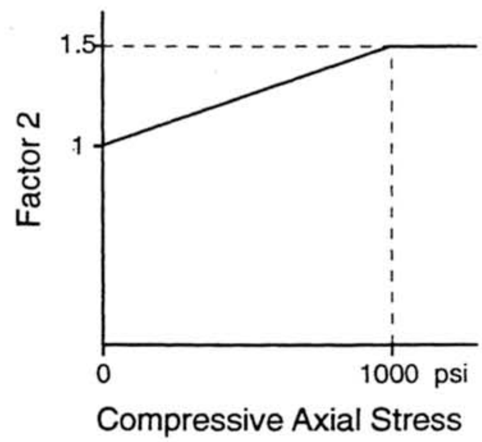


Figure B8

When evaluating existing columns for shear, it is important to note:

1. Concrete is allowed no shear capacity when the column's net axial load is in tension.
2. No reinforcing steel shear capacity is assumed for non-continuous round hoops.
3. No reinforcing steel shear capacity is assumed for rectangular ties when spacing is at 12" (or greater), except where a dense pattern of crossties is present, or 135° seismic hooks are used to close the perimeter ties.
4. Reinforcing steel shear capacity for lap-spliced spirals is taken into account even though it does not meet current practice (welded splices and seismic hooks).

4. Design of Cap Beam and Outrigger for Torsion

Torsion is mainly a problem in outriggers connected to columns with top fixed ends. However, torsion can also exist in bent cap beams susceptible to softening due to longitudinal displacements. This softening is initiated when top or especially bottom longitudinal reinforcement in the superstructure is not sufficient to sustain flexural demands due to the applied plastic moment of the column. Retrofit solutions should ensure adequate members' strength along the load path from superstructure to column foundation.

Caltrans design philosophy is to force column yielding under earthquake loads. In the case of an outrigger, the torsional nominal yield capacity should be greater than the column flexural plastic moment capacity. Torsion reinforcement shall be provided in addition to reinforcement required to resist shear flexure, and axial forces. Torsion reinforcement consist of closed stirrups, closed ties or spirals combined with transverse reinforcement for shear, and longitudinal bars combined with flexural reinforcement. Lapped-spliced stirrups are considered ineffective in outriggers, leading to a premature torsional failure. In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another. Where necessary, mechanical couplers or welding should be used to develop the full capacity of torsion bars. When plastic hinging cannot be avoided in the superstructure the concrete should be considered ineffective in carrying any shear or torsion. Regardless where plastic hinging occurs, the reinforcement for torsional moment strength, T_s, shall not exceed four times the concrete torsional moment strength T_c. In addition, the transverse shear reinforcement strength V_s shall not exceed $8\sqrt{f'_c}\, bd$. Therefore, proportioning outrigger dimensions and reinforcement under combined torsion and shear should obey the above rules described in ACI 318 regardless whether concrete shear and torsion capacities are ignored or not. Prestressing shall not be considered effective in torsion unless bonded in the member.

Unbonded reinforcement however can be used to supply axial load to satisfy shear friction demands to connect outrigger caps to columns and superstructure. Bonded tendons should not be specified in caps where torsional yielding will occur. Designers must consider effects of the cyclic axial load in caps due to transverse column plastic hinging when satisfying shear and torsion demands.

5. *Pipe Seat Extender Vertical and Transverse Capacities*

The capacity of an 8" XX strong pipe seat extender was tested at an undemolished section of the Cypress Street/I-880 viaduct in Oakland. Hydraulic jacks were used to separate the adjoining spans while displacement transducers measured the incremental separation between two spans. Vertical dead load was incrementally applied to the superstructure at different hinge displacements. The test showed a load of 240 kips @ 5.2 in. displacement (i.e., reported force 720 divided by 3 pipes). The pipes were ultimately failed in shear under the 720 kips and at an 8 in. extension after approximately one-third of each pipe section had been cut away. There was no sign of serious straining or distress in the concrete on the anchored end under the 720 kips load at an extension of 8 in. Figure B9 shows a typical hinge detail of the Cypress structure. Results from strain gauges attached to the pipe proved to be not conclusive{1}.

Reference {2} is a study on the vertical and transverse capacities of a pipe seat extender on the Southern Viaduct. Figures B10 and B11 show typical hinge details on the Southern Viaduct. The hinge allows full extension length of 8 inches. Of the six box girder cells, two cells contain two restrainer units each, for a total of four pipes for the entire box girder. The following loading limits were recommended for each 8" XX strong pipe:

$$|\text{Transverse Shear}| + |\text{Vertical Shear}| < 210^{K}$$

$$|\text{Transverse Shear}| < 180^{K}$$

$$|\text{Vertical Shear}| < 180^{K}$$

It is important to mention that vertical shear limitation takes into account full hinge extension while transverse shear limitation is for only 2 in. extension. A design load factor of 1.3 is recommended to account for mis-alignment of up to $\pm 1/8$" (i.e., total misalignment between two pipes $1/4$") because of field installation tolerances.

As discussed above, both the experimental and the analytical investigations report capacities greater than Caltrans recommended value of 100 kips. However, it is still Caltrans policy to use 100 kips per pipe as a desired design value unless space limitations exist. Consulting with SASA/ SEITECH is deemed quite important when capacities higher than 100 kips are used in Design. Furthermore, it is necessary to evaluate the capacity of the supporting hinge diaphragm which could be the limiting load factor. The designer must also evaluate the logistic of inserting the pipe through a limited soffit opening, especially in shallow superstructures.

6. *Column Bar Development Length*

On all new construction and seismic retrofit projects the new ACI 318-89 code shall be used to determine bar development length, except:

> On seismic retrofit projects where it is determined in an Earthquake Retrofit Strategy meeting that the current Bridge Design Specifications guidelines for bar development length can be used.

On structures being retrofitted, post-tensioned bars through the bent cap that provide 250 psi of confining pressure in the area of the column core shall be considered adequate to allow the designer to use the confinement reduction factor ($\phi = 0.8$) which is allowed in either code.

The designer is reminded to determine whether the steel is Grade 40 or 60. If undeterminate, the designer must consider the grade which produces the greatest demands and the least capacity scenarios.

7. *Column Removal/Replacement Falsework Design*

When column removal/replacement is used as a retrofit solution, special consideration for falsework must be given. The falsework design requirements for lateral shear capacity should be related to the existing column shear capacity (minimum 50%), but not less than 0.25g or the maximum expected ground surface acceleration at the site, whichever is less. The stiffness of the shoring should be not less than 50% of the existing column stiffness. Positive connections must be ensured at the shoring top and foundation to mobilize shear and stiffness properties. Friction can be relied on to fully or partially sustain the shear force through the connection provided expected shoring settlement will not compromise the friction force. Horizontal and vertical alignment of the structure must be retained. Joint moment release, created by column removal, must be prevented by strategically locating appropriate shoring under bents and mid-span, etc., of the bridge.

8. *Design/Specification Coordination Issues*

a. Welding Grade 60 to Grade 40 bars.

The designer should be responsible for checking that specifications cover welding of Grade 60 to Grade 40 bars. In retrofit of foundations, outriggers, etc., where welding of Grade 60 to Grade 40 bars is chosen over mechanical couplers, the designer has to ensure that welding is performed with heat specified for Grade 60 bars and the rod is specified for Grade 40 bars. Welds should not be located in potential plastic hinge locations and should be preferably staggered 5' where possible.

b. Roughened concrete surface in shear friction design.

The designer must determine whether it is absolutely necessary to use a higher coefficient of friction of 1.0λ in shear-friction design where concrete is placed against a hardened surface that is intentionally roughened (B.D.S.8.15.5.4.3). This criteria is achieved by attaining a ¼" roughness amplitude. A "¼ inch roughness" is very difficult to define in the specifications and measure in the field and therefore should be avoided. However, if the decision is made to use the ¼" amplitude, the designer should work with the specifications writer to accomplish the transition from design to a field operation. The Specifications Section should produce an SSP that mechanically provides an equivalent ¼" amplitude roughened surface, perhaps in terms of an operation which assumes compliance without heavy reliance on the field engineer's interpretation. The designer may wish to choose an intermediate surface roughness and friction coefficient.

c. Grouting of cored holes with inserted bolts.

When bolts are inserted in cored holes through columns, caps, etc., an installation/ Grouting procedure must be specified to ensure that the desired post-tensioning stress is reached (i.e., bolts are sealed or greased prior to grouting to ensure adequate post-tensioning of bolts). The designer must allow for such factors as steel casing deformation, bolt elastic length, etc.

d. Column removal/replacement shoring.

Details for plan and specification language must be coordinated between the designer and specification writer to ensure there are no duplication, conflicts and omissions.

e. Removal and replacement of restrainers.

The designer must alert the specification writer if restrainers will be removed and replaced, or temporarily disconnected. Some level of restraint must remain in place. Work must be staged to meet this requirement. The Specification Section has a standard SSP, but the designer must determine that it meets the existing conditions.

9. Footing Retrofit Considerations

The designer must perform a complete design when enlarging an existing footing in plan dimensions and depth. An appropriate detail must be shown for chipping the lower corner away to expose reinforcement. Adequate room for welder work space must be provided. Remember, many footings were placed neat (i.e., concrete placed against undisturbed soil) and could have a significant amount of extra cover than what plan details show. Work with the specification writer to provide a contingency plan. Designing the dowel shear connectors on the vertical (shear friction) and horizontal (shear flow) surfaces will require a roughness assumption. If ¼" amplitude criteria is used, mechanical roughening will be required. The top overlay needs to provide sufficient confinement against pullout of column bars. The overlay span between perimeter ties to the bottom mat will determine the thickness, number of reinforcement mats, and rebar size and spacing. Excavation and backfill quantities must be provided, and, perhaps, budget allowances may be required for contracts to provide temporary shoring in the excavation.

10. Pile Foundations

Existing piles must be examined for tension and compression capacity in combination with new perimeter piles. If capacity is exceeded for either condition, the piles must be ignored in the analysis. Of course, a pile failing in tension may still be useable in compression. Piles may fail in tension due to the connection to footing, insufficient tensile reinforcement, or inadequate friction resistance in the soil. End bearing piles will provide little or no tensile resistance.

For piles in soft or liquifiable soils, the designer must consider lateral displacement problems in addition to the vertical load problems. The piles must be evaluated for shear and flexural ductility capacities for the lateral displacements. The P-Δ effects must not be ignored.

For piles in dense, granular soils, lateral resistance will most likely be provided by footings retrofit to resist column plastic hinging. However, the designer should not ignore this check in loose soils. Furthermore, existing footings which don't need full retrofitting should be investigated for capacity to resist lateral demands. Many existing footing/pile systems have deficiencies which will prevent sufficient resistance to demands. The Cypress pile tests (reference 12 in Attachment A) provides some guidelines for steel pile lateral resistance in dense granular soil.

11. Pile Extension Bents

Many of the old pile extension bents are non-ductile and probably don't meet current design standards for columns on pile shafts. Any bridges with pile bents and having multiple simple spans or an intermediate hinge are probably vulnerable to collapse. P-Δ is usually a serious contributor to overload. Shear walls or added pile foundations may satisfy the problems.

12. Exposed Bent Caps

Bridges having exposed bent caps supporting girders on bearings have shown distress in minor earthquakes. The caps routinely crack at the bottom edge where it frames into columns. The positive moment reinforcement is generally insufficient to resist lateral seismic loads. In addition, these caps have joint shear and confinement problems similar to outrigger bents. Transverse prestressing can solve transverse beam moments. However, other retrofit features will be required to solve the array of shear, confinement, longitudinal moment, and possibly torsion problems. The attachment of restrainer cables could be the cause of localized overstresses. Isolation can be a solution to most of the stated problems.

13. Pier Wall System Transverse Design

Design of pier palls in the transverse direction must be consistent with analysis. If the pier is assumed fixed for moment transversely in analysis, this condition must be assured by design. That means the pile connection to the wall or footing must meet the elastic moment and shear designs. This is usually not available in existing pier systems. An alternative might be to allow lateral and rotational springs at the wall base. The lateral springs must represent the sliding friction of wall on sheared piles. The rotational spring must represent the rocking action of the wall on the piles (i.e., a lifting force/displacement iterative process). Of course, once the wall is decoupled from the piles transversely, the longitudinal releases must be modeled consistently with this condition.

14. Seismic Anchor Slab

The designer should be fully aware of the following items when using the seismic anchor slabs as part of their retrofit strategy:

a. The seismic anchor slabs resist both longitudinal and transverse seismic displacements at each abutment.

b. Modelling techniques require "engineering judgement" for the determination of "realistic" abutment springs. Non-linear behavior of the soil and the CIDH piles should be considered when determining longitudinal, transverse, and torque abutment springs. Each abutment should be evaluated for compression effect (i.e., bridge moving towards fill—anchor slab and abutment diaphragm activate large soil wedge) and tension effect (i.e., bridge moving away from fill–anchor slab is dragged across fill). The designer will require soil design parameters from a geotechnical engineer for existing abutment soil conditions.

c. Appropriate attachment details to the existing bridge should be designed at each abutment. The existing structure capacity should be checked for transferring the seismic abutment forces.

d. For anchor slab details, the existing girder layout and/or abutment skew may control CIDH pile and anchor slab trench beam layout.

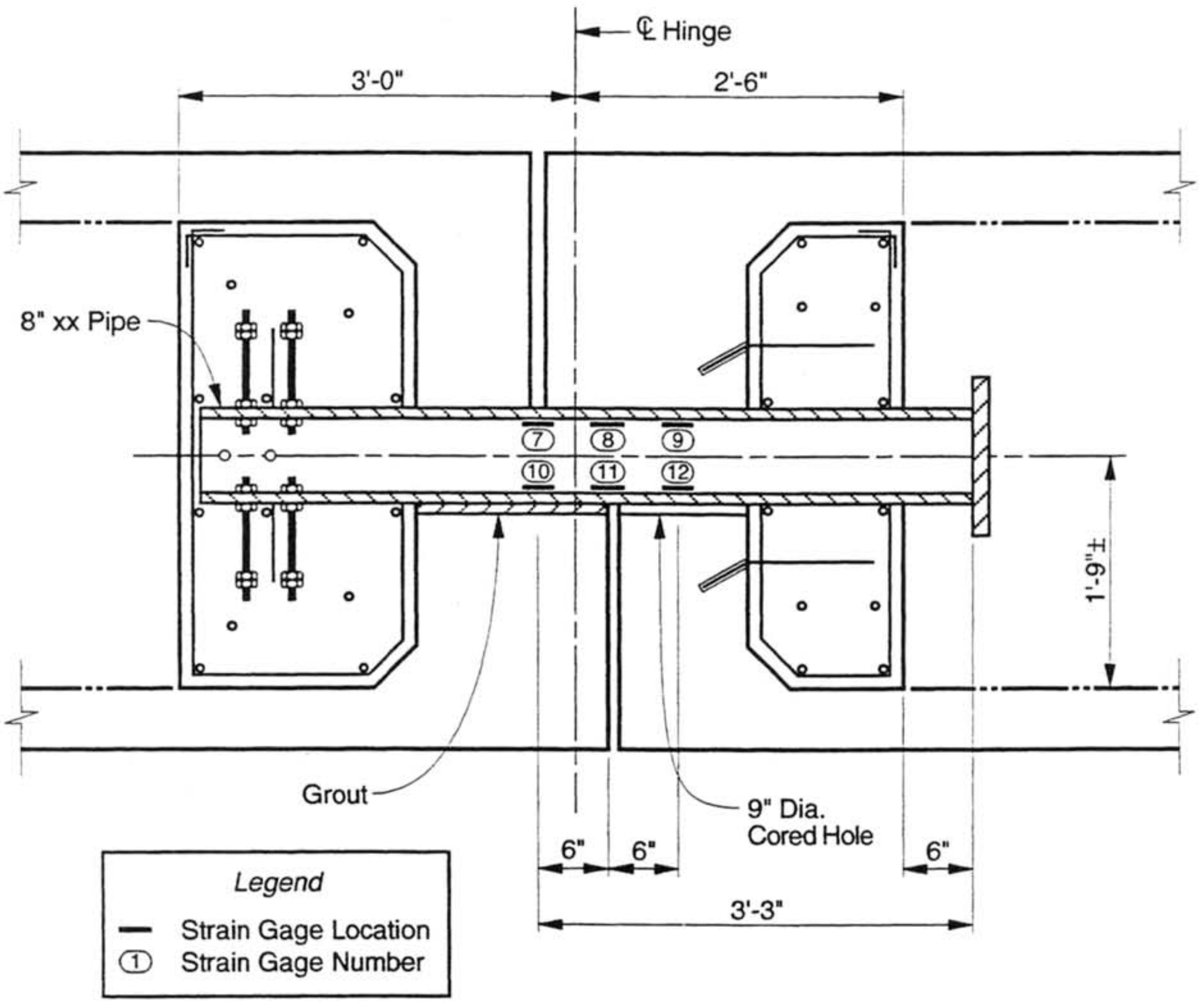
e. Additional damping effects with the seismic anchor slab retrofit should be included in the dynamic analysis. Damping of at least 10% would be expected and a 20% reduction in seismic forces would be appropriate.

f. The designer should consider traffic and utility conflicts early on in the retrofit strategy process. The District or Local Agency should be made aware of all traffic and utility impacts as early as possible. The seismic anchor slab detail could be constructed in stages to minimize the impact on traffic.

References

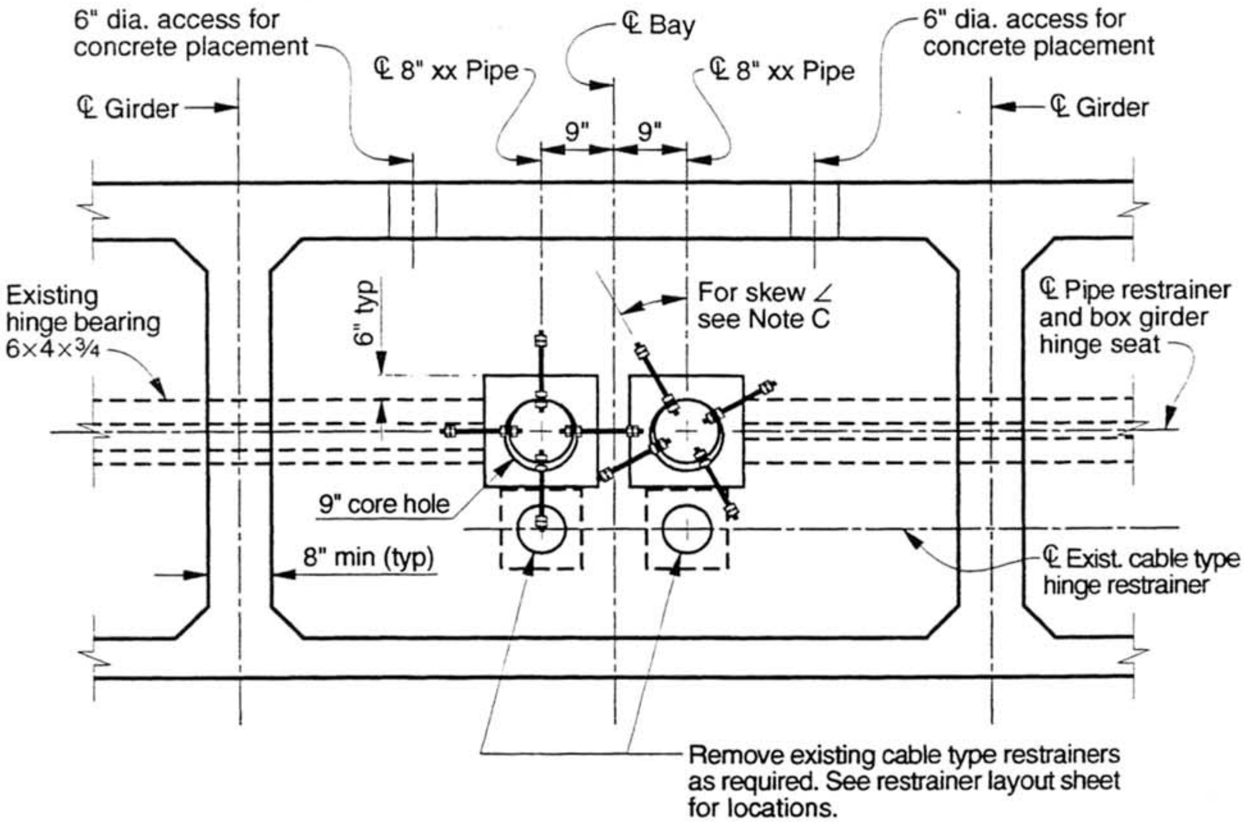
1. "Cypress Street/I-880 Tests of an 8" XX Strong Hinge Pile Seat Extender," report submitted by the Office of Transportation Materials and Research, March 1990.

2. "Ultimate Load Analysis of a Southern Viaduct Hinge Pipe Restraint," report submitted by ANATECH, September 1991.

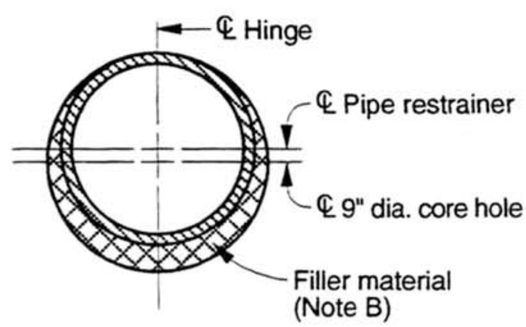


Cypress Street/I-880 Viaduct
Restrainer Test (Strain Gage 7–12)

Typical Hinge Detail on Cypress/I-880 Viaduct
Figure B9

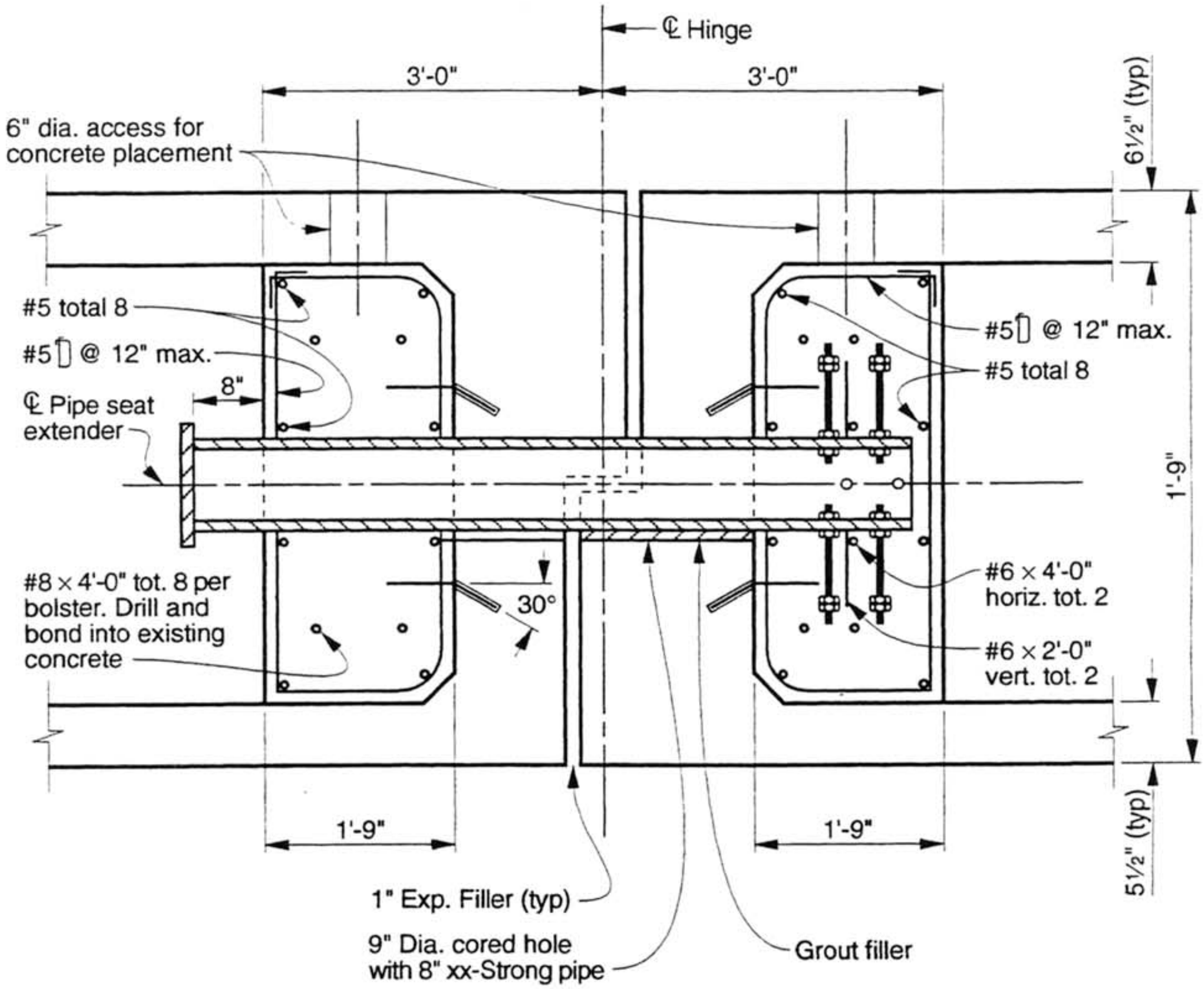


Typical Interior Girder
(Not to Scale)



Section Through Pipe in Grouted Hole
(Not to Scale)

Typical Girder on Southern Viaduct
Figure B10



Typical Hinge Detail on Southern Viaduct

Figure B11